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Peter Rimbos

Ti Huang

COMMONWEALTH OF PENNSYLVANIA
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STATE OF THE ART REPORT ON PRESTRESS LOSSES IN POST-TENSIONED MEMBERS

by
Peter Rimbos
Ti Huang

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Prestress Losses in
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16. Abstract

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Leo D. Sandvig - Director
Wade L. Gramling - Research Engineer
Kenneth L. Heilman - Research Coordinator

Project 74-3: Prestress Losses in Post-Tensioned Members

STATE OF THE ART REPORT ON

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by

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ABSTRACT

This report presents a survey of the literature concerning the prestress loss characteristics of post-tensioned concrete structural members. Findings by previous researchers related to time-dependent loss components (such as creep and shrinkage of concrete and relaxation in steel), the effect of temperature, and additional losses due to post-tensioning (namely friction and anchorage take-up) are presented and discussed. Specification and code provisions currently in use are reviewed, and the inadequate nature of such provisions is cited. The need for further research is indicated in the area of prestress losses in post-tensioned concrete structural members.

1. INTRODUCTION

1.1 Background

In recent years prestressed concrete has rapidly developed into an important structural system. One of the major problem areas in the analysis and design of prestressed members is the proper estimation of the loss of prestress. The fact that prediction methods currently in use are not fully satisfactory has prompted extensive research into prestress losses at Lehigh University.

Fritz Laboratory Project 339 (PennDOT Research Project 66-17), "Prestress Losses in Pretensioned Concrete Structural Members", was begun in October 1966. It's purpose was to develop a rational procedure for the prediction of prestress loss in pretensioned members. Concrete and steel specimens were tested for their elastic, shrinkage, creep and relaxation characteristics. The subsequent analyses of the data led to the development of stress-strain-time surfaces for both concrete and steel and a rational prediction procedure for pretensioned members[24]. The project was completed in September 1973.

A second project undertaken by Lehigh University in the area of prestress loss estimation, Fritz Laboratory Project 382 (PennDOT Research Project 71-9), "Evaluation of Prestress Loss Characteristics of In-Service Bridge Beams", was started in September 1971. This project involved a field study on an experimental pretensioned prestressed concrete bridge in order to evaluate the previously developed prediction procedure (FL 339) and make suitable modifications, if necessary. Comparison between predicted concrete strains and observed values on the test bridge beams indicated that there was good agreement.

The third project of this group FL 402, (PennDOT 74-3), "Prestress Loss of Post-Tensioned Members", began in May 1974 and is estimated to continue for approximately thirty months. The primary objective of this project is to expand the scope of the previously developed prediction procedure to include post-tensioned members. Also, several parameters not considered previously will be studied in this project, they include environmental conditions, the gaining of concrete strength with time, etc.

The first phase of this project is a survey of the literature in order to determine the present state-of-the-art of prestress loss prediction for post-tensioned members. The result of this survey is presented herein.

1.2 Sources of Prestress Loss

1.2.1 General

Loss of prestress is affected by various material characteristics as well as fabrication procedures. The type and strength of both the concrete and steel have an effect on the loss. The significance of the various components of loss depends on whether pretensioning or post-tensioning is employed.

The initial prestressing force is imparted to the steel tendon by the jacking force. Immediately, stress in the tendon begins to diminish with time with an ever decreasing rate. Finally, after maybe several years, the stress will reach a near stable level, this is referred to as the effective prestress.

Prestress losses are known to be caused by: relaxation of prestressing steel, creep of concrete, shrinkage of concrete, elastic shortening of concrete, anchorage slip, and tendon friction. These components will be discussed separately in the following subsections.

1.2.2 Components of Prestress Loss

(1) Relaxation of Prestressing Steel

Relaxation is defined as the stress loss in the steel held under constant strain (constant length). The amount of relaxation for a given steel is a function of the initial stress, temperature, and load duration. Commonly relaxation loss is expressed as a fraction of initial stress at a specified temperature and time. The rate of relaxation tends to increase with increased initial stress and elevated temperatures, but tends to decrease with time. While relaxation theoretically refers to a constant strain condition, the experience of prestressing elements in an actual structural member is not that way. The steel strain gradually decreases on account of creep and shrinkage of concrete, relieving stresses, and reducing relaxation loss to a lower level.

(2) Creep of Concrete

Creep of concrete can be defined as the gradual increase of deformation under a sustained stress. This time-dependent length change varies with the type of aggregate, strength of concrete, stress level, loading age, curing and storage conditions.

Many methods currently exist for the determination of creep. Most commonly the creep strain is estimated as a certain multiple of the elastic strain. The multiplier (creep coefficient) being dependent on various factors. However, there is no universally accepted method for estimating the creep coefficient. Thus estimations of creep may differ much in magnitude.

(3) Shrinkage of Concrete

Shrinkage of concrete is a time-dependent length change caused by chemical and physical changes in the concrete mixture, and the moisture loss in the concrete itself. When the moisture content of the concrete is reduced shrinkage occurs. Small member size, high water content of the concrete mix, and environmental conditions including high temperature, low relative humidity, and poor curing methods all tend to increase shrinkage. Unit shrinkage strain, together with the modulus of steel are commonly used to predict shrinkage losses.

(4) Elastic Shortening

Upon the transfer of the prestress force to the member, there is an accompanying elastic shortening. The deformation is a function of the concrete stress and its modulus of elasticity. Elastic shortening is a more pronounced phenomenon in pretensioned members, where the concrete is bonded to the steel before stress transfer and the concrete is at a lower strength at transfer. In contrast, for post-tensioned members, concrete shortens as the tendons are stretched. Elastic loss is present only when tendons are not stretched simultaneously. In such cases, each tendon, after tensioning and anchoring, suffers an elastic loss as each subsequent tendon is tensioned. Elastic loss is easily calculated on a theoretical elastic basis. For post-tensioned members, it can be either neglected or compensated for by over-stretching, depending upon its magnitude.

(5) Anchorage Slip

In post-tensioned members a decrease of prestress takes place as the jack is released and the tendon force is transferred to the anchor. The anchorage system deforms during this transfer and the tendons slacken. This loss is directly proportional to the anchorage deformation.

The slippage distance is a characteristic of the anchorage system, and is independent of the member design. Consequently, the prestress loss due to anchorage slippage is smaller for a long member with well-lubricated tendons, and vice versa. Friction between the tendon and the surrounding material usually is present when slipping begins, causing a greater anchorage loss near the end of the tendon.

(6) Tendon Friction

At the time of tensioning there exists contact friction between the tendon and the surrounding material thus causing prestress to decrease away from the jacking end. Post-tensioned members will experience a larger loss due to tendon sliding over the entire length. Tendon friction is usually very small in pretensioned members and therefore is commonly neglected.

The friction loss can be considered in two parts. First, the length effect is that amount of friction due to the unintentional out-of-alignment of the tendon frequently referred to as the wobbling effect. The second part of the loss is the curvature effect resulting from the intended tendon curvature.

The loss is dependent upon the coefficient of friction between the concrete materials and the pressure exerted by the tendon on the concrete. Friction in the jack and at the anchorages are additional causes of friction losses. The common methods used to estimate the losses due to tendon friction incorporate the angle through which the tendon is turned, the coefficient of friction between the duct and tendon, and a wobble coefficient.

1.3 Objectives

The objectives of this survey of the literature are listed below.

1. To determine the present state of the art concerning prestress losses in post-tensioned members.

2. To investigate the basis for the provisions in various specifications currently in use.

3. To identify areas where improvements are needed.

The major objectives of this research project are:

1. To develop a method for prestress loss prediction in post-tensioned, as well as pre-post-tensioned structural members.

2. To study the effect of concrete's gain of strength with time on the prestress loss.

3. To determine the effect of elevated temperatures common to curing on the loss.

4. To determine the effect of relative humidity and other ambient environmental conditions on the overall loss characteristics.

5. To modify the general prediction procedure developed in Fritz Engineering Laboratory Report No. 339 to include the above mentioned parameters.

6. To prepare and document a general computer program for prestress loss prediction for prestressed concrete structural members.

It is expected that this research work will lead to an improved estimate of prestress losses in post-tensioned and pre-post-tensioned prestressed concrete structural members. Consequently, a more accurate control of member performance and improved economy should result.

2. PRESTRESS LOSSES - TIME-DEPENDENT

2.1 General

One of the main problems in the analysis and design of a prestressed member is the proper estimation of the loss of prestress over an extended period of time. Unsatisfactory performance or possibly failure may result from underestimation of losses, while undesirable camber could result from overestimation.

Various design specifications include widely different provisions for prestress loss estimation. At one extreme, some codes simply specify a straight percentage or a constant value (AASHTO[4], PennDOT[34]). In contrast, other specifications include lengthy step-by-step procedures requiring the use of many equations, graphs, and numerical constants (PCI[36]).

This chapter is primarily concerned with the long-term prestress loss components, namely: creep and shrinkage of concrete, and relaxation in steel. These three time-dependent components are very heavily interdependent upon one another and hence cannot be completely separated. Furthermore, accurate estimation of these long-term losses is difficult on account of the multitude of factors affecting them, such as the quality and composition of concrete, the quality of steel, the environment in which the structure is kept, curing process of concrete, etc. A few suggested prediction procedures are briefly described and compared in the following paragraphs.

In 1967 an expression was developed for the loss of prestress by Ghali, Neville and Jha[18]. Elastic and creep recoveries of concrete strain induced by steel relaxation were taken into account, as well as the increase of the modulus of elasticity of concrete with time. The authors believed that previous investigators had ignored at least one of these factors. Consequently, a step-by-step procedure was used incorporating an estimate of the concrete modulus of elasticity, as well as several other assumptions.

In the process the prestress loss at the end of any interval can be calculated, provided that the loss at the beginning of the same interval is known. The loss in any interval is assumed to act from the middle of the interval up to the time at which the loss is required. Each loss component - shrinkage, creep and relaxation - is considered separately for each interval.

In 1973 Ghali and Dilger[19] simplified the previous method[18]. Interaction of the three time-dependent components is accounted for by the use of graphs intended for practical design. Assumptions include a constant modulus of elasticity of concrete (equal to the value at the time of application of prestress) and that prestress is applied at one stage only.

The step-by-step procedure used in developing the graphs and other design aids incorporates the strain compatibility condition that the total strain in the steel and concrete due to all causes must be equal at the end of each interval in order to determine the loss during that interval.

Only the final value of the loss is given by this newer method. It was believed that the final value of loss was sensitive only to the final values of creep, shrinkage and relaxation, not to their variation with time.

In 1970 Sinno and Furr[43] devised a method (to be used only for pretensioning) to estimate the ultimate value of the prestress loss. Their method utilizes a hyperbolic time function for shrinkage and creep, and a simplified two-cycle iteration procedure to account for the variation of concrete stresses.

The two-cycle iterative procedure was actually a variation of the rate of creep method which assumed that the time-dependent components were constant over short time intervals. Sinno and Furr concluded that the complete rate of creep method was too long and complex to use for predicting loss.

Shrinkage and unit creep strains are needed in order to use this method. Shrinkage strains are assumed to be independent of the levels of prestress, while creep strains are dependent on the levels of prestress across the depth of the beam.

It was seen that the time-dependent strains and prestress losses attain considerably slower rates of growth at 100 to 150 days after application of prestress. This age was believed to be dependent on the ambient relative humidity. Also the hyperbolic method yielded prestress loss values only slightly higher than the rate of creep method, thus indicating that the two-cycle iteration was adequate.

Branson[12] conducted a systematic investigation in 1971 at the University of Iowa on the material and structural response of prestressed concrete members and developed a procedure for estimating prestress losses. This method includes equations with separate terms for the prestress loss components. Some component interaction is recognized in the expression for creep loss, but not in any other terms.

Continuous time functions are used for all needed parameters, as well as some approximate equations and several correction factors. All correction factors are applied to ultimate values.

The procedures used by Branson for predicting time-dependent material and structural behavior represent a nominal approach for design purposes. These procedures are neither definitive nor statistical and therefore Branson concluded that probabilistic methods be used for an accurate estimate of the loss.

In 1972 Glodowski and Lorenzetti[20] presented an "interaction" method for prestress loss prediction. The basis of this method is a stress-strain balancing technique in which the service life is divided into small time intervals during which the relaxation, creep and shrinkage can be assumed to be independent of each other.

Since interaction between steel and concrete is considered, a procedure for determining steel stress relaxation losses under conditions of changing strain is established. The procedure developed involves transfer from one stress level relaxation curve to another for each change in strain (refer to Section 2.3 and Fig. 1).

The accuracy of the interaction method is dependent upon the accuracy of the material property determinations, as well as proper description of actual stress levels and environment of the structure during its life.

The interaction method yielded lower total prestress losses than if loss causes were treated independently. The reduction is generally more significant for steel stress relaxation.

In all of the methods covered in this section there was good agreement with experimental results and/or other research findings. However, many of these methods of prediction involve cumbersome calculations and somewhat lengthy step-by-step procedures. As a result, these methods are more suitable for computer solution than for manual application.

2.2 Creep and Shrinkage

2.2.1 General

Creep and shrinkage are two important parameters to be considered in prestressed concrete when estimating prestress loss. Prestressed concrete can be considered as a more general case of reinforced concrete, but the presence of prestress in a member increases the importance of both creep and shrinkage effects. Both these phenomena, as do other losses, reduce the compressive stresses which prestress induces in a member. The serviceability of a member could be impaired by such a reduction in prestress resulting in undesirable formation of cracks.

Although creep and shrinkage phenomena are commonly known, they have not been uniformly defined. Researchers have usually supplied definitions for specific use in their experiments.

Abeles[2] described creep thus: "If the concrete is subjected to any kind of stress for some considerable time, then the initially produced deformation will continue to increase and this increase will go on for years." Abeles also mentions that creep is primarily a permanent phenomenon and attributes it to the plastic properties of the gels contained in the concrete while they are still moist.

Even when no stress is acting, concrete undergoes a gradual deformation with time. This is due to the movement of water from or to the surrounding medium, and is commonly referred to as shrinkage.

For many years the two phenomena, creep and shrinkage, were considered additive. Thus, the total increase in strain was assumed to be the sum of shrinkage (equal in magnitude to that of an unstressed member) and of a change in strain due to stress (creep). This method proved suitable for those applications where both creep and shrinkage occurred simultaneously. More recently, shrinkage and creep are found not to be independent, and therefore not additive.

Neville[33] gives definitions for creep of a drying concrete member subjected to a sustained stress, which enhances the understanding of the creep phenomenon. The creep occurring under conditions of no moisture movement into or out of the surrounding medium is termed basic creep, whereas the additional strain caused by the concurrent drying process is called drying creep. Neville mentions that by separating creep into these two components, the real phenomenon can be interpreted correctly. He describes three stages of creep under a constant applied stress: primary (decreasing rate of creep), secondary (steady-state creep), and tertiary (associated with increasing strain rate, cracking and increased actual stress, and finally resulting in failure of material). In prestressed concrete members, however, the concrete stress does not remain constant. Herein lies the difficulty in accurately estimating creep of concrete.

It should be noted that the effect of shrinkage in post-tensioned members is rather small, since an appreciable amount of shrinkage occurs before the prestress is applied to the member. Similarly, as the prestress is usually applied at a later stage, the loss due to creep is also smaller than that in a pretensioned member. Consequently, a smaller loss is frequently allowed for in post-tensioned members.

2.2.2 Influencing Factors

There are many factors known to influence creep and shrinkage of concrete members. In ACI Special Publication - 27, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures" (1971)[8], the principal variables affecting both creep and shrinkage, in most cases, are listed. The variables considered were: time of initial loading and time initial shrinkage is considered, environmental humidity, minimum thickness of member, water-cement ratio in the form of slump and cement content, and mix proporitions in the form of percent fines and air content. Most researchers generally agree with the above listing.

In accordance with the factors mentioned above, ACI also presented several correction factors to account for deviations from a carefully chosen "normal" condition. These factors are normally not excessive and tend to offset each other. In general, for design purposes, these may normally be neglected with the possible exception of the effect of member size and slump.

In 1964 Neville and Meyers[32] listed the following factors as affecting creep: age of member, elastic modulus of the aggregate used, relative humidity, drying process, moisture movement, and temperature. As was previously explained, the effect of creep is greatest and therefore of largest importance at an early age. At later ages, however, Neville and Meyers asserted that the rate of creep becomes independent of the ambient relative humidity, and that shrinkage is essentially completed.

Podolny[37] in 1969 explained that more creep will occur when the member cross-section is small, the concrete mix used is relatively weak (high water-cement ratio), and the air is dry in the member environment. In fact a joint committee of ACI-ASCE[6] stated that creep strain under dry conditions may be as much as three times the creep strain occurring in humid conditions.

Shrinkage is known to be dependent on time and existing moisture conditions, but not on stresses. The amount of shrinkage in a concrete member is somewhat proportional to the quantity of water incorporated into the mix. Weather condition is the most important factor influencing the rate of shrinkage.

In Leonhardt's[26] studies it is shown that more shrinkage will occur in a warm, dry environment. Rich concrete mixes will have relatively greater shrinkage than those having high water-cement ratios. A concrete member will experience more shrinkage when exposed to drying early than if the concrete were allowed to harden for a longer time. Also, because they dry out rapidly, small members will incur greater and faster shrinkage than larger ones.

In 1964 Hanson[21] completed a three-year investigation of the effect of accelerated curing conditions on the creep and shrinkage properties of concrete typically used in precasting plants. Hanson found that there was a beneficial reduction of prestress loss due to creep and shrinkage by use of accelerated curing methods. The studies were based on the assumption that creep and shrinkage are independent in nature which introduces only minor errors in stress prediction.

It is known that in order to provide the cement with enough water to harden, the concrete must be kept damp for the very early part of its life. This process is commonly referred to as curing. Curing or maturing improves the final strength, particularly at the surface. Hanson observed that the use of atmospheric steam curing reduced the creep, shrinkage and prestress loss by as much as 40% of values obtained using normal curing techniques. When he tested specimens cured under a high pressure type of curing called autoclave curing, the creep and shrinkage were almost negligible.

Hanson concluded that the prestress loss due to creep in post-tensioned members varied from 6 to 12% of the initial steel stress under normal curing conditions depending on the type of aggregate and cement used in the mix. Also he concluded that the loss due to shrinkage over long periods of time may be two and one-half to three times that due to creep over a similar period of time.

2.2.3 Creep Expressions

Creep strain is intimately related to, and is often estimated as a multiple of, the instantaneous elastic strain. In order to properly evaluate creep we must know the variation of the modulus of elasticity of concrete with time and an expression for creep.

Two broad types of creep expressions exist[33]. One type includes those expressions which tend to a limiting value: exponential and hyperbolic. The other type consists of expressions which increase indefinitely: power and logarithmic. The latter is often used up to an arbitrary time.

Many exponential equations of creep take the following form (Thomas, McHenry, Arutyunyan, Lyse, L'Hermite):

$$c = F(k)\ [1 - e^{-At}] \qquad (2\text{-}1)$$

where c = creep strain

$F(k)$ = a function of age at loading, ultimate creep strain

A = constant

Other authors have proposed hyperbolic expressions which enable one to predict the ultimate creep value rather quickly and easily from experimental data. Ross[33] presented the following expression:

$$c = \frac{t}{A + Bt} \qquad (2\text{-}2)$$

where A and B are constants.

It can be easily seen that the ultimate creep is equal to 1/B. Hyperbolic expressions have been found to closely fit experimental values and are frequently used[12,43].

Power and logarithmic expressions are not commonly used to predict creep, though the U. S. Bureau of Reclamation[32] has developed a logarithmic expression which has agreed with long-term experimental data quite well:

$$c_{sp} = F(k)\ \ln\ (t + 1) \qquad (2\text{-}3)$$

where c_{sp} = specific creep (creep per unit stress)

$F(k)$ = an experimental parameter respresenting the rate of c_{sp} with logarithm of time

All of the above mentioned expressions for creep can be used only when test data is available to evaluate time-functions and parameters, etc. When no creep tests are made the literature suggests the use of creep prediction charts[33]. In 1937 Ross formulated a chart to evaluate a given hyperbolic expression for creep. More recently, Jones and Wagner developed sets of charts utilizing a standard creep curve based on "normal" conditions. However, both of these charts have limitations; the former only applies to concrete moist-cured for seven days, while the latter does not consider the aggregate properties.

Three broad methods exist for computing creep under variable stress: effective modulus method, rate of creep method, and the principle of superposition. In using these methods the specific creep is used.

The effective modulus method uses a reduced or effective modulus which allows for both elastic and creep strain. The method disregards the stress history in that the strain at any time is assumed to depend upon the stress at that time only. Acceptable results are obtained so long as the applied loads do not vary very greatly[25].

In the rate of creep method the time rate of specific creep is assumed independent of the previous stress history and is subsequently integrated over the time since first loading. Consequently the stress history is taken into account to some extent, but this only applies to increases in stress.

The third method of predicting creep under variable stress is the principle of superposition. Each stress increment is thought to produce a resulting deformation component continuing for an infinite time. The effects of all stress changes are then summed. However, this method requires the use of creep-time curves for the various ages at loading at which an increment of stress is applied.

It should be mentioned that expressions for shrinkage prediction parallel quite closely those for creep prediction, and hence are not discussed separately in this report. This topic is reviewed in ACI SP-27[8] which contains several papers on shrinkage, as well as creep prediction.

2.3 Relaxation in Steel

As pointed out earlier, relaxation refers to stress loss in the steel without strain changes. Most of the reported relaxation studies referred to tests under the "constant length" condition. Under this strain condition, it is generally accepted that the total amount of stress relaxation for a given steel is a function of the initial stress, temperature and the stressing procedure. It should be pointed out that the conditions of the prestressing steel in a prestressed concrete member is somewhat more complicated. The steel strain gradually decreases with time on account of creep and shrinkage of the concrete. On the other hand, the steel strain could increase with an addition of load to the member.

In the paragraphs which follow, the work of four separate researchers will be described. It should be emphasized that in all cases, the basic tests used by the researchers were under the constant length condition. For application in the analysis of prestressed concrete members, the basic relaxation expressions were extended to deal with varying strain conditions. The extension is discussed in Section 2.3.2.

2.3.1 Constant Length Tests

Magura, Sozen and Siess[30] presented a paper describing a comprehensive research program on the stress relaxation of prestressing wires, which was concluded in 1964. Their studies included the relationship between relaxation loss and the initial stress level and time, as well as the effect of prestretching. In addition, they made an extensive review of relaxation information available in literature.

Based on their own tests as well as data from previous investigators, Magura et al. arrived at the following conclusions:

1. The significance of relaxation loss of prestress lies in its effect on the service performance of the structural member. Therefore, the estimate of loss should not be viewed as an end by itself, but a means to estimate the prestress remaining.

2. Relaxation loss is strongly influenced by the initial stress-yield strength ratio (initial stress ratio). For initial stress ratios less than 0.5, relaxation loss may be neglected.

3. The effect of pre-stretching on relaxation loss is insignificant if the pre-stretching time is short.

4. Within the ordinary temperature range, the effect of temperature on relaxation loss is negligible. Using all of the avail-data, Magura et al. derived the following expression relating the remaining stress at any time to the initial stress and yield strength of the steel

$$\frac{f_s}{f_{si}} = 1 - \frac{\log t}{10}\left[\frac{f_{si}}{f_y} - 0.55\right], \text{ for } \frac{f_{si}}{f_y} \geq 0.55 \qquad (2\text{-}4)$$

where f_s = remaining stress at time t after prestressing

f_{si} = initial stress in steel

f_y = yield strength of steel, defined by the 0.1% offset

t = time, in hours

Podolny and Melville[38] presented a paper describing the several variables affecting relaxation losses in prestressing steel wire. Their study, concluded in 1969, involved an investigation of research concerning the relaxation characteristics of steel. As a result of the study, Podolny and Melville reached the following conclusions: "At normal initial stress levels and temperatures, relaxation is predictable and is of relatively minor significance in terms of other losses imposed upon the structure or member. At elevated temperatures, the losses due to relaxation of the prestressing steel is of greater significance ..."

In 1970 Glodowski and Lorenzetti[20] presented a paper dealing with a method for prestress loss prediction. These authors reviewed the literature and concluded that Magura's straight-line, semi-log expression (Eq. 2-4) was not sufficiently accurate for short periods of time less than 1000 hours. This conclusion is understandable in light of the fact that Magura utilized relatively long-term data in deriving the stress relaxation expression.

Glodowski and Lorenzetti conducted their own relaxation tests on prestressing steel wires, and developed a method for extrapolating short-term data to predict steel stress relaxation losses at much longer times. Their steel relaxation expression takes the following quadratic form.

$$\%\ SR = A + B\ (\ell n\ t) + C\ (\ell n\ t)^2 \qquad (2\text{-}5)$$

where $\%\ SR$ = precent stress relaxation

t = time in hours

A, B and C are functions of the initial stress ratio

As in Magura's expression, Eq. 2-5 predicts higher relaxation for higher initial stress ratios. The authors showed that the quadratic expression was quite accurate for short times, and also fairly consistent with other methods of long-term stress relaxation prediction.

Relaxation properties of prestressing strands were studied at Lehigh University over a five year duration[11,41]. The tests involved seven-wire stress-relieved strands of the 270 K grade, under a constant length condition. The controlling variables studied were type and size of strand and the initial stress level.

In the Lehigh study, particular attention was paid to the selection of suitable time functions so that an extrapolation of long-term relaxation loss for periods of up to 100 years would be feasible. The investigators developed the following expression for the prediction of relaxation loss as a function of initial stress and time

$$\%\ SR = \frac{f_{si}}{f_{pu}} [B_1 + B_2 \log (t + 1)] + \left(\frac{f_{si}}{f_{pu}}\right)^2 [B_3 + B_4 \log (t + 1)] \quad (2\text{-}6)$$

where $\%\ SR$ = relaxation loss, in % of initial stress

$\frac{f_{si}}{f_{pu}}$ = initial stress, in fractions of guaranteed ultimate strength

t = time, in days

B_1, B_2, B_3 and B_4 are constants which depend on the type and size of strand.

The form of this expression very closely resembles the quadratic equation developed by Glodowski and Lorenzetti. The only difference comes in the latter's $(\ell n\ t)^2$ term. In fact Batal and Huang[11] examined a three-term, quadratic logarithmic function, but found this function to be overly sensitive to variations of data at the beginning and also at the end of the testing period. In view of this, they eliminated this function from further long-term investigation[41].

2.3.2 Effect of Varying Strain

In actual prestressed concrete members there is an interaction between steel and concrete, hence conditions of changing strain in steel are always prevalent. Glodowski and Lorenzetti[20] established a procedure for determining steel stress relaxation under these conditions. As mentioned in an earlier section, they developed a "curve transfer" procedure to account for varying strain. Curves corresponding to different initial stress levels were first constructed, showing percent steel stress relaxation vs. time. For each change in strain, a transfer is made from one stress level relaxation curve to another, by maintaining the same percent loss (horizontal transfer). Relaxation loss during the next time interval is then estimated following the new curve (see Fig. 1).

In 1974 the PCI Committee on Prestress Losses[36] suggested a modified form of Magura's expression to be used in a step-by-step calculation. The expression is used over a finite time interval with the steel stress at the beginning of the interval substituted for the initial stress. The expression takes the following form

$$RET = f_{st}\left[\frac{\log 24t - \log 24t}{10}\left(\frac{f_{st}}{f_{py}} - 0.55\right)\right], \text{ psi} \qquad (2\text{-}7)$$

where $(f_{st}/f_{py} - 0.55) \geq 0.05$ and $f_{py} = 0.85\ f_{pu}$

RET = relaxation over time interval t_1 to t, psi

f_{st} = steel stress at t_1, psi

f_{py} = 0.1% offset stress, psi

t = time at end of time interval, days

t_1 = time at beginning of time interval, days

In comparison with the Glodowski-Lorenzetti procedure, it can be seen that PCI's method involves an "equal time" (vertical) transfer between relaxation curves (refer to Fig. 1).

In order to facilitate the solution of the problem of the time dependent variation of strain, Schultchen, Ying, and Huang[41] developed the concept of a stress-strain-time relationship. Geometrically speaking, their expression describes a three-dimensional surface in a stress-strain-time coordinate system. The contour lines parallel to the stress-time plane indicate the gradual decrease of stress under a constant length condition. The expression is as follows:

$$f_s = D_1 + D_2 s + D_3 s^2 - s\,[C_1 + C_2 \log (t + 1)] - s^2\,[C_3 + C_4 \log (t + 1)] \qquad (2\text{-}8)$$

where f_s = remaining stress in the strand

s = steel strain in in/100 in.

t = time in days

$D_1, D_2, D_3, C_1, C_2, C_3, C_4$ are constants which depend on the type and size of strand.

The above expression was derived from relaxation test data where the strain was a time-independent constant. However, it was assumed to be also directly applicable to cases when strain varies with time. The use of this expression was found to result in an upper bound solution for the stress remaining, and therefore, a lower bound for the loss value. The applicability of the three-dimensional surface is restricted to initial stress values in the range of 0.5 f_{pu} to 0.8 f_{pu}. Also, time values may vary from approximately one day to 100 years.

To summarize, at the present time, researchers continue to experiment on the relaxation behavior of steel (usually strands). The phenomenon of steel stress relaxation has been described and its characteristics experimentally varified. Researchers have identified the various factors affecting relaxation behavior. Expressions have been developed for prediction of stress loss due to relaxation (constant length). In addition, the actual phenomenon of a varying strain has been investigated by several researchers and expressions have been formulated to help predict the stress loss under these conditions. Most of the proposed expressions agree with the test data used by the particular investigators.

2.4 Temperature Effects

In recent years, the interest in the temperature effects on prestress loss characteristics has been stimulated by the greater use of prestressed concrete pressure vessels in nuclear reactors. As a result, most of the recent research has concerned itself with high temperatures (up to 1000° F). This range is well above the range commonly experienced by prestressed concrete highway structural members, approximately -20° to 40° C (-4° to 104° F). Hence the conclusions from these investigations may not be directly applicable to the problem under consideration.

Experimental evidence has shown that the rates of creep and shrinkage are increased by an increase in temperature. These increased rates may result in either greater deflections and displacements or internal stress changes, depending on whether or not movement is allowed in the structure. Hence, temperature should be thought of as an influential parameter in accurately identifying the time-dependent effects in prestressed concrete members.

In 1962 England and Ross[15] carried out tests on 4-1/2 in. x 12 in. cylinders in order to determine the strain in heated concrete due to loading. Auxiliary tests were made to separate the effects of concrete expansion due to a rise of temperature and contraction due to shrinkage.

Observing that the time-dependent strains were greatly temperature-dependent, England and Ross concluded that at elevated temperatures (up to 140° C), creep may be several times greater than at normal temperatures. It was also observed that for sealed cylinders there is a more significant effect in the range of 20-60° C (68-140° F) than in the range of 100-140° C (212-284° F).

In a later paper (1965), Ross, England, and Suan[40] reviewed their studies on prestressed concrete beams under a sustained temperature crossfall. They indicated that in the presence of a non-uniform state of temperature, a redistribution of stress will occur in the member, since creep is time-dependent. Also, using their previous data[15], they assumed that creep is approximately, directly proportional to the temperature in °C.

In relation to shrinkage, England and Ross[15] noted that when a concrete specimen is heated it will lose more water in reaching moisture equilibrium than an unheated specimen. Since shrinkage is associated with loss of moisture they concluded that shrinkage is increased by heating.

In 1967 Arthanari and Yu[10] conducted tests indicating that the rate and magnitude of creep increases much with the temperature of a specimen. Also, the specific creep showed a nearly linear variation with temperature under both uniaxial and biaxial systems of loading.

Further investigation indicated higher creep values when the temperature was increased in steps under constant stress, than when the maximum temperature was continuously maintained. However, Arthanari and Yu concluded that it may be necessary to conduct more tests under controlled conditions for these results to be more conclusive. Their reason for caution was that at elevated temperatures between the range of 20° and 100° C, the nature of creep was highly unpredictable.

Neville[33] mentioned that creep is not a "monotonic function" of temperature. In tests conducted by Nasser and Neville it was thought more convenient to relate temperature to the increase in creep beyond twenty-one days after loading, because of some irregularities encountered in the first few days after loading. They concluded that the rate of creep reaches a maximum in the vicinity of 160° F (71° C) when it is approximately 3.5 times the rate at 70° F (21° C). Beyond this temperature the creep is reduced and the rate of creep drops off steadily. At 205° F (96° C) the rate of creep is almost the same as at about 125° F (52° C).

In relation to the temperature effects on the relaxation behavior of prestressing steel, it is known that elevated temperatures cause relaxation loss to increase. In 1974 Schwier and Hofmann[42] conducted several tests on the relaxation characteristics of steel wires and strands after an eight hour heat treatment at 80° C. As expected, the short heat treatment resulted in an increased relaxation loss during the heated period. However, after cooling to room temperature, unloading and subsequently reloading to a higher initial stress than before (19.8 ksi, higher than the eight hour stress at 80° C), the subsequent relaxation took a significantly slower course than the relaxation of a steel not so heat treated.

Through extrapolation of their results for 1000 hours, Schwier and Hofmann concluded that for longer times (say one to one and one-half years), the remaining stress in the steel will be similar to a steel not so heat treated. Beyond this time, the heat treatment could have beneficial results on the relaxation characteristics of the prestressing steel.

In addition to oil-hardened, drawn, and stress-relieved steels, Schwier and Hofmann tested 1/2 in. stabilized strand (≃ 280 K). This steel behaved differently than the others tested in that the stress remaining in the steels under both testing procedures (short heat treatment at 80° C and room temperature), converged in about nine days. This is primarily due to the way in which stabilized strand is processed.

In conclusion, since investigators do not wholly agree on the temperature effect on the relaxation of steel, further experiments are necessary so that a better picture of this effect may be obtained. However, it can be readily seen that in the normal range of temperatures encountered in highway structures, the time-dependent components of prestress loss, namely creep and shrinkage of concrete and relaxation of steel, are only slightly affected by temperature. Consequently, only very simple modifications need be made (if any) to the normal methods of predicting prestress loss.

3. POST-TENSIONED PRESTRESS LOSSES

3.1 General Remarks

Anchorage and friction losses are predominent in post-tensioned members. Friction loss occurs at the time of tensioning. For a pretensioned member, this loss is negligibly small and occurs only if the tendons are deflected. Consequently, post-tensioned members will suffer greater friction loss as the tendons slide against the concrete over the entire length of the member. When tendon force is transmitted from the jack to anchor, a slight deformation of the anchorage system will occur causing a loss of such force. Since this loss of prestress is caused by a fixed total amount of shortening, the percentage of loss is higher for short tendons than for long ones. Consequently, post-tensioned tendons suffer a greater loss than the tendons of a pretensioned member.

The theoretical methods employed to calculate anchorage deformation and consequently loss due to anchorage take-up are clearly defined and well accepted. Anchorage slip is simply a function of the load in the tendon and the type of anchorage used. The percentage loss of prestressing force is a function of length and will be higher for short lengths commonly the case in post-tensioned members.

The generally accepted procedures for determining the losses due to friction and anchorage slip are presented in the following sections.

3.2 Friction Losses

As discussed earlier contact friction will exist along the tendon at the time of initial tensioning causing prestress to decrease away from the jacking end. The two parts of the friction loss are the unintended wobbling effect and the intended tendon curvature.

The basic relationship for the loss of prestress due to friction is[5]:

$$P_x = P_s\ e^{-(\mu\alpha + Kx)} \tag{3-1}$$

where
- μ = curvature friction coefficient
- α = angular change of direction of tendon, in radians
- K = wobble friction coefficient, in units per foot
- x = distance from jacking end to any point on tendon, in feet
- P_s = steel force at jacking end
- P_x = steel force at a point away from the jacking end.

The coefficient of friction depends primarily upon the smoothness and nature of the contact surfaces. Whereas, the stress in the tendon and the total angular change along its length determines the pressure existing between the tendon and concrete. The length and stress of the tendon, as well as the friction coefficient, also influence the wobbling effect of the duct.

The derivation of the basic relationship for friction loss is based on some simple assumptions and approximations. First, the values for K and μ are about the same for both tensioning and detensioning of the steel, this being true for smooth wires having smooth-walled sheaths. T. Y. Lin presents a complete theoretical analysis for friction loss[28].

3.3 Anchorage Take-Up

3.3.1 General

Anchorage take-up refers to the loss of prestress force upon the release of the jacking device. This loss is due to the deformation or slippage of the anchoring device.

The generally-accepted formula used for computing the loss of prestress due to anchorage deformation at transfer is

$$\Delta f_s = \Delta_B E_s / L \tag{3-2}$$

where Δ_B = deformation of anchorage and/or slippage of wires

L = length of tendon over which anchorage loss is distributed

The above formula assumes no friction during such slippage. However, in most post-tensioning systems this is not the case, as friction will almost always exist between the tendon and the surrounding material. When friction is considered, the anchorage loss becomes non-uniform, with larger losses near the ends of the member.

Wang in an article entitled "Loss of Prestress Due to Anchorage Take-Up"[45] considers the effect of friction between the steel tendon and its surrounding material during detensioning of the steel at the time of anchorage take-up. He examines two cases: whether or not the overtensioning necessary to overcome friction loss exceeds the loss of stress due to anchorage take-up. If it does, the tendons are first overtensioned and then the steel stress is released back to the required value necessary for anchoring. Whereas in the opposite case, the jacking stress must be increased in order to attain the required value for anchoring.

As Lin[28] explains overtensioning is only one of the methods used to help overcome the frictional loss in the tendons. Another means of reducing this loss is to jack from both ends. This method is used quite often when tendons are long, even though it does involve more work in the field. The use of lubricants to reduce friction is still another alternative.

A moderate friction loss can usually be compensated for by over-tensioning. If friction is excessive, over-tensioning becomes infeasible, and other means must be used. It is important to note that the overtension required to overcome friction is not cumulative over that required for overcoming anchorage losses or for minimizing relaxation in steel. This is true since for all of these cases the overtensioning simply consists of an overstretching and a subsequent release-back.

Heinen in "Post-Tensioning with Thread Bars"[22] considers another aspect of anchorage loss. He cites that wedge-type anchorages have disadvantages other than slip. High stresses in the wedges and their surroundings frequently exceed the yield point. Therefore, plastic deformation of the wedge creates additional slippage after some time. However, this slippage cannot be overcome by overstressing of the tendons and hence a loss of prestressing force takes place at the anchorage.

Heinen further states this problem is of minor importance when the anchorage is at the end of the member. Only when coupling of such tendons is incorporated will this plastic slippage be of major concern since it will cause cracking to occur. In order to avoid the aforementioned disadvantages, prestressing tendons called "threadbars" have been developed in Europe. It is possible to screw on the bars and anchorages which should reliably develop the full strength of the bar. Therefore threadbars can be cut and subsequently anchored at any point desired. Heinen hopes that these new (pre 1969) prestressing tendons will eliminate elastic as well as plastic slippage in anchors and couplings.

3.3.2 The Anchorage Loss Phenomenon

Anchorage seating loss has been studied by many individuals and a definition of this phenomenon can be easily obtained.

Huang, in his article "Anchorage Take-Up Loss in Post-Tensioned Members"[23], describes the mechanism behind anchorage loss. During the tensioning operation friction causes a gradual decrease in tendon stress away from the jacking end. Upon release of the stress, anchorage deformation causes the tendon to slide inward at this end. In this vicinity friction reverses its direction thereby reducing the tendon stress at the end. The tendon reaches a maximum value at the point where the inward movement is stopped and prestress at this point is not changed upon anchorage. Consequently, the loss of prestress due to anchorage take-up will gradually increase from zero at this point to a maximum value at the end.

The phenomenon just described represents the prestress variation along the tendon. This variation is characterized by the frictional parameters of the prestressing system. If these parameters are known along with the anchorage take-up distance, then the length of the back sliding segment and, thus the stress loss at any point, can be calculated. (The general relationship for friction loss was previously described, Eq. 3-1.)

Huang[23] further notes that the anchorage loss at the end may be many times larger than the average loss for the full length of the tendon. In addition, back slip could take place over more than half of the tendon when post-tensioning from one end only.

Turula and Freyermuth in "Moment Influence Coefficients for Continuous Post-Tensioned Structures"[44] discuss losses due to, what they refer to as, "anchor set". Two equations are presented for calculating the loss of prestress force (ΔP_o) at the anchor section. The first equation is

$$\Delta P_o = 2\sqrt{rAE\Delta L} \qquad (3\text{-}3)$$

where r = loss of prestress force per unit length of beam (computed from a straight line approximation of ACI's exponential relationship, previously cited)

A = cross-sectional area of prestressing tendons

E = elastic modulus of the prestressing tendons

ΔL = tendon movement at the anchor due to "anchor set"

The above formula applies only if

$$\Delta P_o \leq 2\ (P_o - P_{MIN}) \qquad (3\text{-}4)$$

where P_o = jacking force

P_{MIN} = lowest computed prestress force in beam (located at the non-jacking end for jacking done from one end or near midspan for jacking from both ends)

Another case, which is infrequently encountered, occurs when the anchorage loss is distributed throughout the entire tendon length. A trapezoidal distribution of stress versus distance (from the jacking end to the centerline of the member) is thus formed. For this case the following formula was suggested

$$\Delta P_o = P_o - P_{MIN} + \frac{rAE\Delta L}{P_o - P_{MIN}} \qquad (3\text{-}5)$$

The loss of prestress force given by Eq. 3-5 will always be greater than the value obtained from Eq. 3-3. This case was not dealt with by the previous authors cited and it should be noted that it would only occur when anchorage deformation is excessive.

The equations presented by the authors cited herein are very similar, and in some cases identical. Also, the anchorage loss phenomenon has a firm theoretical basis as the investigations have been carried out independently of each other.

4. PRESENT SPECIFICATIONS

4.1 General

As previously pointed out, precise determination of stress losses in prestressed concrete members is a complicated problem because of the varied interaction between the time-dependent loss components. It is, strictly speaking, impossible to separately estimate the several loss components, particularly in view of the wide variations of prestress, environment, and loading conditions.

It should be emphasized that an error in computation of losses can affect the service performances such as deflection and cracking, but has no significant effect on the ultimate flexural capacity for bonded members.

In the following sections, the methods used in several specifications and codes for prestress loss prediction in post-tensioned members will be presented. All of these methods deal specifically with the ultimate loss of prestress, with the exception of the PCI Committee on Prestress Losses' tentative recommendations[35]. This method uses a step-by-step procedure and is therefore capable of predicting the loss at any time.

A term by term comparison among the several methods is not attempted. On account of the differences in which the component interactions were considered in the several methods, the direct comparison among terms will not be meaningful.

4.2 AASHTO Specifications[4]

Loss of prestress due to all causes may be determined by the following method. Other methods may be used as long as they are supported by the appropriate research data.

Total loss, excluding friction, is expressed as the sum of the four major components:

$$\Delta f_s = SH + ES + CR_c + CR_s, \text{ psi} \tag{4-1}$$

Shrinkage:

$$SH = 0.80\ (17{,}000 - 150\ RH) \tag{4-2}$$

where RH = average annual ambient relative humidity, %

(value taken from a figure-map of United States)

Elastic shortening:

$$ES = 0.5\ \frac{E_s}{E_{ci}}\ f_{cir} \tag{4-3}$$

(certain tensioning procedures may alter this value)

where E_s = modulus of elasticity of steel (28×10^6 psi)

E_{ci} = modulus of elasticity of concrete at transfer of stress ($33\ w^{3/2}\ \sqrt{f'_{ci}}$, common formula)

f_{cir} = concrete stress at center of gravity of the steel due to prestress and dead load immediately after transfer; computed at section(s) of maximum moment. (At this stage the initial stress has been reduced by the elastic shortening of the concrete and tendon friction.)

Creep:

$$CR_c = 12\ f_{cir} - 7\ f_{cds} \qquad (4\text{-}4)$$

where f_{cds} = concrete stress at c.g.s. due to all dead load except dead load present at time the prestress is applied.

Relaxation:

$$CR_s = 20{,}000 - 0.3\ FR - 0.4\ ES - 0.2\ (SH + CR_c) \qquad (4\text{-}5)$$

(only for 250 - 270 k strand)

where FR = friction loss stress reduction below the initial stress level of 0.70 f'_s at the point under consideration. Post-tensioned tendons are commonly stressed in excess of 0.70 f'_s to allow for anchorage seating loss. Due to this overstressing it is necessary to limit the value of FR in the equation to the reduction below 0.70 f'_s, rather than the <u>total</u> friction loss, in order to retain the presumption of the equation that the initial stress is 0.70 f'_s. (calculated by Eq. 3-1, Section 3.2)

f'_s = ultimate tensile strength of steel

In lieu of the preceding method, the following estimates of total losses, excluding friction, may be used under normal conditions:

Type of Prestressing Steel	Total Loss, psi	
	f'_c = 4,000 psi	f'_c = 5,000 psi
Wire of strand	32,000	33,000
Bars	22,000	23,000

The lump sum loss values may be used for bridges with concrete strengths 500 psi above or below the values of 4000 psi and 5000 psi listed in the table headings.

4.3 ACI Code

The ACI Code itself[5] lists the causes for the loss of prestress. For an explanation on how to compute these losses, two references are given[6,7]. The methods presented in these references make up the following:

Friction: refer to Eq. 3-1, Section 3.2

Elastic shortening:

$$\text{loss} = n\Delta fc \qquad (4\text{-}6)$$

where $n = \dfrac{E_s}{E_c}$, modular ratio

Δf_c = average concrete stress along one tendon from end to end of the tendon caused by subsequent tensioning

Shrinkage:

shrinkage strain $\simeq$ 0.0002 ~ 0.0003

Creep:

creep strain = 100% ~ 300% of elastic strain, for very humid and very dry conditions, respectively

Relaxation:

loss ≃ 2 ~ 8% of initial steel stress

Two alternate methods are also permitted[6,7].

The first method should be used when the individual losses may be predicted with reasonable accuracy.

$$\Delta f_s = (u_s + u_e + u_d)\, E_s + \delta_1 f_{si} + \delta_2 f_{si} \qquad (4\text{-}7)$$

where u_s = shrinkage strain

u_e = elastic shortening strain

u_d = creep strain

δ_1 = ratio of loss in steel stress due to relaxation

δ_2 = ratio of loss in steel stress due to friction during prestressing

f_{si} = initial stress in steel after seating of anchorage

The second alternate method is to be used when specific loss data are lacking (does not include friction loss).

$$\Delta f_s = 25{,}000 \text{ psi} \qquad (4\text{-}8)$$

4.4 PennDOT Specifications[34]

The PennDOT Design Manual incorporates a modified version of the Bureau of Public Roads (1955) formula for loss of prestressing force with a minimum specified loss of 20% for box beams and 22.8% for I-beams.

For pre-post-tensioned members the total design loss for the pre-tensioned portion is:

$$\Delta f_{si} = 6000 + 16\ fcs + 0.08\ fsi \qquad (4\text{-}9)$$

For the post-tensioned portion the following formula is used:

$$\Delta f_{si} = 3000 + 11\ fcs + 0.04\ fsi \qquad (4\text{-}10)$$

where f_{si} = initial tensile stress in prestressing steel, usually 70% of ultimate, psi

f_{cs} = concrete stress at c.g.s. at time of release, psi

In both these equations care must be exercised in computing the concrete stress at release. The concrete stress used in Eq. 4-9 is based on the center of gravity of the pretension tendons. For Eq. 4-10 the center of gravity refers to the post-tensions tendons. The initial steel stress of each equation refers to pretension and post-tension tendons, respectively.

Design curves have been developed for pretensioned members based on the modified Bureau of Public Roads formula, Eq. 4-9. Similar design aids are yet to be developed for pre-post-tensioned members.

The PennDOT Design Manual does not specifically address itself to post-tensioned members. Nevertheless, it seems reasonable to assume that Eq. 4-10 can be used for post-tensioned members as well.

4.5 PCI Tentative Recommendations[36]

The PCI Committee on Prestress Losses has only recently issued the following tentative recommendations for estimating prestress losses of post-tensioned members.

Total loss is expressed in the following fashion:

$$TL + FR + ANC + ES + \sum_{t_1}^{t} (CR + SH + RET), \text{ psi} \qquad (4\text{-}11)$$

Friction: FR; refer to Eq. 3-1, Section 3.2

Anchorage: ANC

"Slip at the anchorage will depend upon the particular prestressing system utilized and will not be a function of time. Realistic allowance should be made for slip or take-up as recommended for a given system of anchorage."

Elastic Shortening:

ES will be based on the modulus of elasticity of concrete at the time of transfer. "The average concrete stress between anchorages along each element shall be used in calculating shortening."

Time-Dependent Losses:

"Prestress losses due to steel relaxation and creep and shrinkage of concrete are interdependent and are time-dependent. To account for changes of these effects with time, a step-by-step procedure with respect to time shall be used with the time interval increasing with age of the concrete. Shrinkage from the time curing is stopped until the time when the concrete is prestressed shall be deducted from the total calculated shrinkage for post-tensioned construction. It is recommended that a minimum of four time intervals be used as follows:"

Step	Time Interval* Beginning, t	End, t
1	End of curing	Age at transfer
2	End of step 1	Age = 30 days or time of loading
3	End of step 2	Age = 1 year
4	End of step 3	End of service life

* Note: When significant changes in loading are expected, time intervals other than those recommended should be used.

Creep:

$$CR = (UCR \times SCF \times MCF)\ (PCR)(f_c),\ \text{psi} \qquad (4\text{-}12)$$

where f_c = net concrete compressive stress at c.g.s. at time t_1 taking into account the loss of prestressing force occurring over the preceding time interval.

UCR = ultimate loss due to creep (psi of stress loss for each psi of compressive stress in the concrete)

$$= A - \frac{20\ E_c}{10^6} \geq 11 \text{ psi} \qquad (4\text{-}13)$$

where A = 95 psi, for moist cured not exceeding seven days for normal weight concrete

= 76 psi, for moist cured not exceeding seven days for lightweight concrete

= 63 psi, for accelerated cured concrete for both normal weight and lightweight

SCF = a factor that accounts for the effect of size and shape of a member on creep

(value found in table on page 10 for different volume-to-surface ratios)*

MCF = a factor that accounts for the effect at age of prestress and length of moist cure on creep

(only for moist cured concrete; table, page 11)

PCR = amount of creep over time interval t_1 to t

= (AUC at t) - (AUC at t_1)

where AUC = amount of ultimate creep at time after prestress transfer (table, page 11)

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* Tables and page numbers in this section refer to TENTATIVE RECOMMENDATIONS FOR ESTIMATING PRESTRESS LOSSES by the PCI Committee on Prestress Losses.

Shrinkage:

$$SH = (USH \times SSF)\ (PSH),\ \text{psi} \tag{4-15}$$

USH = ultimate loss due to shrinkage, psi

$$= 27{,}000 - \frac{3{,}000\ E_c}{10^6} \geq 12{,}000, \text{ for normal weight concrete}$$

$$= 41{,}000 - \frac{10{,}000\ E_c}{10^6} \geq 12{,}000, \text{ for lightweight concrete}$$

SSF = a factor that accounts for the effect of size and shape of a member on shrinkage (value found in table on page 12 for different volume-to-surface ratios)

PSH = amount of shrinkage over time interval t_1 to t

$$= (\text{AUS at } t) - (\text{AUS at } t_1) \tag{4-17}$$

where AUS = amount of ultimate shrinkage at time after end of curing (table, page 13).

Relaxation:

$$RET = f_{st}\left[\frac{\log 24\ t - \log 24\ t_1}{10}\left(\frac{f_{st}}{f_{py}} - 0.55\right)\right],\ \text{psi} \tag{2-7}$$

(only for stress-relieved steel)

where $(f_{st}/f_{py} - 0.55) \geq 0.05$ and $f_{py} = 0.85\ f_{pu}$

f_{st} = steel stress at time t_1, psi

f_{py} = 0.1% offset, psi

f_{pu} = guaranteed ultimate tensile strength, psi

for low relaxation steel: replace 10 with 45; use

$f_{py} = 0.90\ f_{pu}$, not $0.85\ f_{pu}$

4.6 Summary

As can be surmised from the preceding sections, the inadequate nature of current specifications is particularly true for post-tensioned prestressed concrete members. The procedures given for post-tensioned members are, in many instances, only slight modifications of the prescribed pretensioned procedures.

Only recently have post-tensioned members been used in greater numbers in the United States. As a result there has been little study on post-tensioned structural systems in this country. In view of the substantial increase in applications of structures of this type, better methods are needed for estimation of prestress losses of these members.

5. CONCLUSIONS

As a result of the survey of the literature on prestress losses in post-tensioned concrete structural members, presented herein, the following conclusions have been made:

1. Many of the prediction methods presented by researchers involve long procedures compounded, in some cases, by complicated calculations.

2. For environmental and loading conditions normally encountered by bridge members both creep and shrinkage are well defined phenomena. Also, since the influence of the various factors on creep and shrinkage are qualitatively and, for practical purposes, quantitatively known, expressions have been developed to aid in determining the losses caused by them.

3. The relaxation of steel stress under normal conditions has also been adequately described and experimentally proven. The phenomenon of varying strain actually encountered in practice has been investigated. Though the methods incorporated to predict the behavior of the member under this condition are vastly different, the end results are essentially the same.

4. In the normal range of temperatures the time-dependent components of prestress loss are only affected to a small degree, (creep, shrinkage and relaxation all increase with a rise in

temperature). As a result, modifications of normal prediction procedures can be made rather easily.

5. Expressions used to determine the losses due to friction and anchorage slip have a strong theoretical basis and are well accepted.

6. The interaction among the several components of prestress loss still needs to be investigated further. The current specifications used for post-tensioned members give adequate procedures for determining the prestress loss. In many cases the pretensioned procedures are simply modified for use in post-tensioned prediction. Also, while pretensioned procedures have been adjusted with the advent of new research, post-tensioned procedures have not been changed (i.e. PennDOT).

Since post-tensioned structural systems have only recently come into more widespread use in the United States, past study on post-tensioned members is minimal. Also, more sophisticated and realistic methods are needed in order to adequately predict the prestress loss in post-tensioned members. The improved economy, safety and serviceability, which would surely result from such research, would result in wider applications of post-tensioned members in the construction of many new highway systems.

6. ACKNOWLEDGMENTS

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7. FIGURE

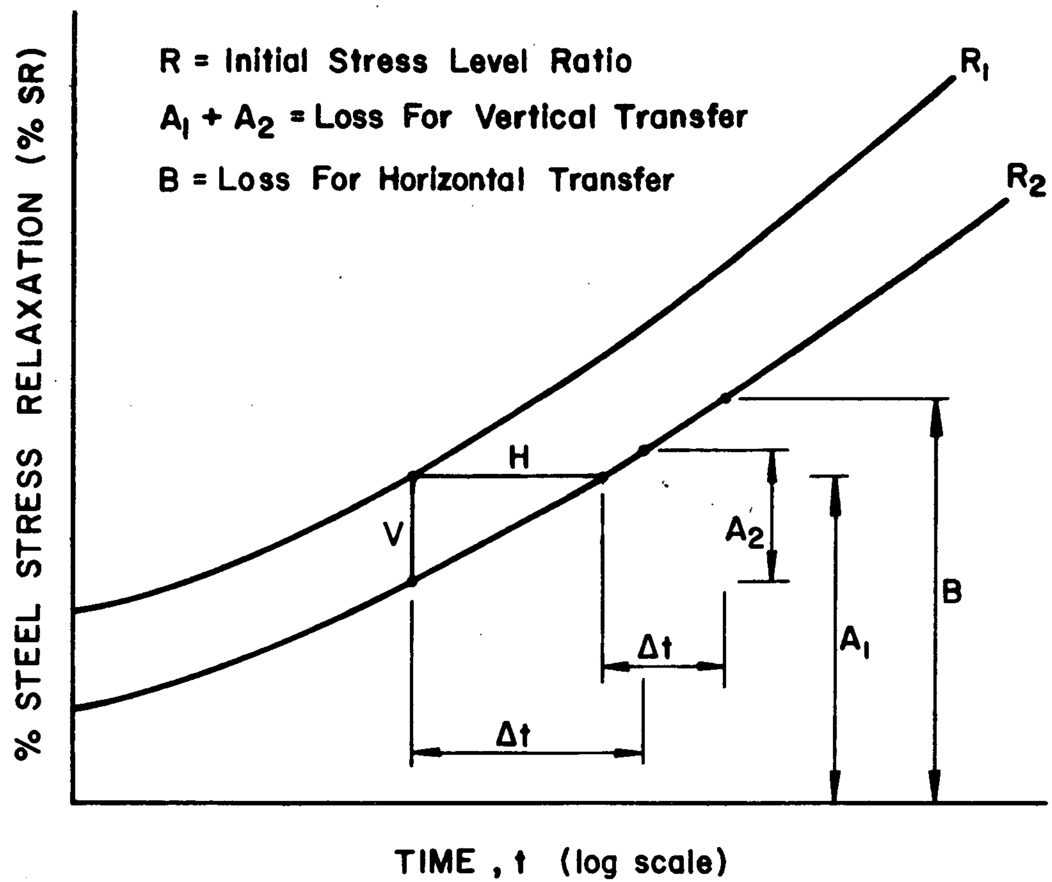

Fig. 1 Effect of Curve Transfer Methods[20]

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